TENSION FIELD ACTION IN PLATE GIRDER UNDER VARIOUS LOADING CONDITIONS

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ABSTRACT

In general plate girders are designed using two different theories: design using beam theory and tension field theory. Until the latest revision of IS 800-2007 tension field theory was not popular in Indian Engineering Industry. But the present code allows the user to adopt the tension field theory for girders in engineering construction. This paper is focusing on analytical investigation on the ultimate load behavior of beam member designed using tension field theory. The frame is subjected to different load conditions. A parametric study was carried out with slenderness ratio of the web as parameter. The present study concluded that the member with slenderness ratio 150 to 175 can be used in seismic prone areas, as it have more reserve strength due to post buckling and tension field action.

Key words: Plate Girder, Loading Conditions, Indian Engineering Industry.


1. INTRODUCTION

Plate girders are normally designed to support heavy loads over long spans in situations where it is necessary to reduce an efficient design by providing girders of high strength to weight ratio. The search for an efficient design procedure conflicting the requirements, particularly in
the case of the web plate. To produce the lowest axial flange force for a given bending moment, the web depth must be made as large as possible. To reduce the self weight, the web thickness must be reduced to minimum. As a consequence, in many instances the web plate is of slender proportions and is therefore prone to buckling at relatively low values of applied shear.

IS 800-2007 clause 8.4.2.2 offers two shear buckling design methods for the design of plate girders having slender web panels. The two methods are,

- Simple post-critical method, which may be applied to both stiffened and unstiffened girders and is therefore of general application.
- b) Tension field method, which may only be applied to girders with intermediate transverse stiffeners. Even for such girders its range of application is limited to a range of stiffener spacing defined by: 1.0 ≤ c/d ≤ 3.0. The simple post-critical method is seen as a general-purpose method which can be applied to the design of all girders. The tension field method can be applied to a certain range of girders only, but will lead to considerably more efficient designs for these girders, because it takes full account of the post-buckling reserve of resistance. The value of the tension field stress at which yield will occur, termed the “yield strength of tension field ” in IS 800-2007, may be determined by applying the Von Mises-Hencky yield criterion.

Alinia et al. (2009) modelled and analysed a number of full scale plate girders in ABAGUS. They studied the shear failure mechanism characteristics. The objective of the study was to clarify how, when and why plastic hinges that emerge in the experimental tests actually form. The addition of end-posts provides more fixity to flange plates and increases the ultimate resistance of plate girders. Baskar and Shanmugan (2003) have done experimental investigation on steel-concrete composite plate girders subjected to the combined action of shear and bending. Two bare steel plate girders and six composite plate girders were tested to failure in order to study ultimate load behavior. The ultimate load carrying capacity and the tension field width of composite plate girders are found to increase significantly compared to the bare steel girders. Baskar and Shanmugan (2003) have done experimental and numerical investigation on steel-concrete composite plate girders. Two bare steel plate girders and ten composite plate girders two different web-depth to thickness ratios and varying $M_p/M$ ratios of the flanges were considered in the investigation. They analyzed the behavior of steel-concrete composite plate girders when subjected to shear loading. Attention is focused on the variation in the tension field action in web panels. Extensive strain measurements have been made on the web panels in order to obtain a detailed picture of the tension field action. The width of the tension field and the strength are found to be increased in the steel-concrete composite plate girders under shear loading. Cedric et al. (1988) performed large deflection elastic-plastic analysis in ADINA software and studied influence of flange bending rigidity on shear capacity of plate girder with slender web. Shear capacity is reached when the web yields in shear in tension corner with no diagonal tension for girders with small bending rigidity. On the other hand, girders with impractically heavy flanges after first yielding of web the capacity increases due to tension field action.

Chai and Sung (2006) revisited a fundamental assumption used in most classical failure theories for post buckled web plates under shear, namely that the compressive stresses that develop in the direction perpendicular to the tension diagonal do not increase any further once elastic buckling has taken place. Estrada et al. (2006) tested eight stainless steel plate girders with different aspect ratio and slenderness of the web panel, along with the rigid or non-rigid condition of the end post. Two different behaviours were observed during the first experimental campaign dealing with the effect of the girders with rigid and non-rigid end
post. The plate girder with rigid end post was characterised by the tension field development after the buckling phenomenon, followed by the formation of a sway frame mechanism which meant the collapse of the structure and its ultimate capacity with the excessive yielding of the web panel. Hanbin et al. (2000) studied cyclic elasto-plastic large displacement analysis of steel columns and portal frame. The presented cyclic hysteretic curves have been compared with the experimental results. The experimental and analytical results showed that the horizontal restoring force is reduced by vertical loads. The existence of large vertical load induces significant destabilizing effects in the frame structure. This paper is concerned with an analytical investigation on the ultimate load behavior of beam member of a steel frame designed using tension field theory. The frame is subjected to different load conditions. A parametric study was carried out with slenderness ratio of the web as parameter.

2. MODELING AND ANALYSIS

2.1. Material Properties

The entire plate girder is developed from mild steel plates (Grade 43) of different thicknesses. The average value of yield stress ($f_y$) of the steel is 270 MPa, the Young’s Modulus (E) is 210GPa. And the Poisson’s ratio is 0.3. To perform non-linear analysis, inelastic properties of the material have to be considered. The stress-strain values for steel are shown in figure 1.

![Stress Strain Curve](image)

2.2. Finite Element Method

Abaqus 6.9-1 is used as a tool to perform non-linear analysis in the plate girder. A three dimensional finite element model of the frame is developed. The element type chosen is S4R. It is a 4-node doubly curved general-purpose quadrilateral, conventional stress/displacement. Material property is isotropic linearity for elasticity and for plasticity, isotropic hardening model is used for hardening behavior.

In structural analysis, non linearity is used to describe a problem where the relationship between load and deflection is not constant. This occurs when stiffness of the structure changes over the course of simulation. Two types of nonlinearities are there; geometric non-linearity and material non-linearity. Both nonlinearities have been considered for present study. Eigen value buckling analysis is carried out to incorporate initial imperfections in post buckling analysis. The first mode shape from buckling analysis is considered as initial imperfection and then nonlinear analysis is carried out. A load-deflection (Riks) analysis is used for studying the post buckling behavior.
3. RESULTS AND DISCUSSIONS

3.1. Validation
In order to validate the model developed, reference girders has been taken from literature [3] and analyzed. The mid span deflections under various load steps were monitored and retrieved from the finite element analysis. A graph as shown in Fig.2 was plotted between load and the mid-span deflection and was compared with the experimental prediction. The finite element model was able to predict the ultimate failure load and the behavior of the girder with an acceptable error.

![Validation of analysis procedure](image)

**Figure 2** Validation of analysis procedure

A steel frame is designed using IS 800-2007 with both ends fixed condition. The effect of residual stress is not considered in the analysis. The details of the steel frame used are shown in the figure 3. And dimensions of models taken for parametric study have been mentioned in table 1.

![Details of the Steel Frame](image)

**Figure 3** Details of the Steel Frame
Table 1 Dimensions of the Plate Girder

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Web depth, (d) (mm)</th>
<th>Web thickness, (t_w) (mm)</th>
<th>Flange width mm</th>
<th>Flange thickness mm</th>
<th>(d/t_w)</th>
<th>Aspect ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPG1</td>
<td>750</td>
<td>5</td>
<td>200</td>
<td>20</td>
<td>150</td>
<td>1.5</td>
</tr>
<tr>
<td>SPG2</td>
<td>750</td>
<td>4.2857</td>
<td>200</td>
<td>20</td>
<td>175</td>
<td>1.5</td>
</tr>
<tr>
<td>SPG3</td>
<td>750</td>
<td>3.75</td>
<td>200</td>
<td>20</td>
<td>200</td>
<td>1.5</td>
</tr>
<tr>
<td>SPG4</td>
<td>750</td>
<td>3.33</td>
<td>200</td>
<td>20</td>
<td>225</td>
<td>1.5</td>
</tr>
<tr>
<td>SPG5</td>
<td>750</td>
<td>3</td>
<td>200</td>
<td>20</td>
<td>250</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Material properties have been taken as the same as that of plate girder used for validation purpose. In ABAQUS a mesh size of 30 mm was given in global meshing tool and element type of S4R has been chosen for meshing purpose. Maximum limits for deflection (both horizontal and vertical) has been taken from IS 800:2007.

3.2. Behavior under Vertical Load Alone
Post buckling analysis has been carried out with initial imperfections from buckling analysis and studied the ultimate load behavior for various models of different slenderness ratios. The first yield point load, corresponding deflection at mid span, ultimate vertical load capacity, corresponding deflection and ductility factor for different models are given in the table 2. And vertical load –mid span deflection is given in figure 4.

Table 2 Parametric Study under Vertical Load condition

<table>
<thead>
<tr>
<th>Sl.No.</th>
<th>Web slenderness ratio (d/t_w)</th>
<th>Web thickness, (t_w) (mm)</th>
<th>First yield load (kN)</th>
<th>Mid span deflection at first yield load (mm)</th>
<th>Ultimate vertical load (kN)</th>
<th>Ductility factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>150</td>
<td>5</td>
<td>880</td>
<td>3.6</td>
<td>1172.86</td>
<td>6.94</td>
</tr>
<tr>
<td>2</td>
<td>175</td>
<td>4.29</td>
<td>800</td>
<td>3.9</td>
<td>1028.4</td>
<td>6.41</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>3.75</td>
<td>800</td>
<td>3.8</td>
<td>879.95</td>
<td>6.578</td>
</tr>
<tr>
<td>4</td>
<td>225</td>
<td>3.33</td>
<td>750</td>
<td>3.2</td>
<td>785.83</td>
<td>7.8</td>
</tr>
<tr>
<td>5</td>
<td>250</td>
<td>3</td>
<td>540</td>
<td>2.8</td>
<td>561.7</td>
<td>8.93</td>
</tr>
</tbody>
</table>

![Figure 4 Load-Deflection Behavior Under vertical load](http://www.iaeme.com/IJCIET/index.asp)
It was observed that, when a plate girder designed using tension filed theory is used as a beam member of a steel frame, as the slenderness ratio of the beam increases, its ultimate load carrying capacity reduces. From Table 2, comparing the ultimate load with first yield point load, it is clear that ultimate load is 33%, 28.55%, 10%, 4.7% and 4% more than the yield strength for beams with slenderness ratios 150, 175, 200, 225 and 250 respectively. Beams with comparatively less slenderness ratio have more reserve strength once after yielding due to its post buckling strength and tension field action. Whereas the contribution of tension field action for very slender sections (225 & 250) is negligible. And plate girders with slenderness ratios 150, 175, 200 the post buckling capacity is high and plastic hinge formation is more clear compared to other two models.

Figure 5 (a) Model with loading and boundary conditions in ABAQUS, (b) Buckled shape of SPG1 under lateral load alone, (c) Maximum In-Plane Principal Stress Distribution of SPG1, (d) Von-Mises Stress Distribution of SPG1

3.3. Behavior under Lateral Load
To study the ultimate lateral load carrying capacity and its ductility characteristics of the frame, lateral load alone has been applied. Post buckling analysis has been carried out with initial imperfections proportional to first mode of buckling analysis. The ultimate lateral load capacity and ductility factors for different models are listed out in the Table 3 and lateral load vs lateral deflection is given in figure 6.
Table 3 Parametric Study when lateral load alone acting

<table>
<thead>
<tr>
<th>Sl.No.</th>
<th>Web slenderness ratio d/t_w</th>
<th>Web thickness, t_w (mm)</th>
<th>Ultimate lateral load (kN)</th>
<th>Lateral deflection at first yield load (mm)</th>
<th>Ductility factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>150</td>
<td>5</td>
<td>814.84</td>
<td>16.3</td>
<td>1.22</td>
</tr>
<tr>
<td>2</td>
<td>175</td>
<td>4.29</td>
<td>776.42</td>
<td>15</td>
<td>1.33</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>3.75</td>
<td>741.87</td>
<td>10.84</td>
<td>1.84</td>
</tr>
<tr>
<td>4</td>
<td>225</td>
<td>3.33</td>
<td>712.00</td>
<td>9.9</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>250</td>
<td>3</td>
<td>680.66</td>
<td>8.78</td>
<td>2.28</td>
</tr>
</tbody>
</table>

Figure 6 Load-Deflection Behavior up to a deflection of 30 mm

Figure 7 (a) Model with loading and boundary conditions in ABAQUS, (b) Buckled shape of SPG1 under lateral load alone, (c) Maximum In-Plane Principal Stress Distribution of SPG1, (d) Von-Mises Stress Distribution of SPG1
It is observed that, as the slenderness ratio of the beam increases from 150 to 250, its ultimate lateral load carrying capacity reduces from 814 kN to 680 kN. The structure is ductile with ductility factor ranging from 1.22 to 2.28.

3.4. Behavior under Constant Vertical Load and Gradually Increasing Lateral Load

The structure was loaded with vertical load (30% of ultimate capacity) and an incremental lateral load. For each model, ultimate vertical load alone has been applied with initial imperfections. The Lateral load – deflection curve is shown in Figure 7.

![Graph showing Lateral Load-Deflection Behavior Under Combined Loading Condition](image)

4. CONCLUSIONS

ABAQUS results are compared with already available experimental results and validated. Under vertical load, that ultimate load is 33%, 28.55%, 10%, 4.7% and 4% more than the yield strength for beams with slenderness ratios 150, 175, 200, 225 and 250 respectively. Beams with comparatively less slenderness ratio have more reserve strength once after yielding due to its post buckling strength and tension field action. Under pure lateral loading on frame member, it is observed that, as the slenderness ratio of the beam increases from 150 to 250, its ultimate lateral load carrying capacity reduces from 814 kN to 680 kN. The structure is ductile with ductility factor ranging from 1.22 to 2.28. Under combined loading condition also, the frame shown a ductile behavior. With present study it can be concluded that, the member with slenderness ration 150 to 175 is preferable in seismic prone areas, as it have more reserve strength due to post buckling and tension field action. Further study need to be performed for detailed understanding of the problem with cyclic loading or any other dynamic loading.

REFERENCES


